ABSTRACT

A 1950’s vintage chemical process structure was reviewed in 2012 as part of the post-Fukushima seismic walkdowns and subsequently taken out of service for seismic upgrades. The upgrades included strengthening the building to meet licensing requirements supplemented by current seismic hazard data and a seismic margins assessment to quantify the facilities median seismic capacity.

The six-story braced frame structure was expanded and upgraded numerous times over its 50+ year life. Consequently, the structure has irregular bracing layout, horizontal eccentricities between mass and stiffness, vertical discontinuities in floor elevation and discontinuous floor diaphragms. The building’s irregularities couple the lateral response in the North-South and East-West directions. Additionally, the diagonal bracing in a given direction has different strength depending on the direction of seismic load. Thus, the seismic margins assessment included eight median-based pushover analyses to quantify the building’s median reserve strength. Facility modifications were implemented to support the development of member yield strengths used in the pushover analysis, that ranged from strengthening connections, to reinforcing columns to strengthening floor diaphragms. The pushover results were also used to identify non-yielding members that did not require strengthening.

The reduction in seismic demand due to inelastic energy dissipation is based on EPRI NP-6041-SL. A range of fragility curves are developed and each fragility curve is convolved with the mean seismic hazard to obtain the annual probability of seismically induced failure. This value was reported to the regulatory agency and was a positive factor in the decision to resume operations at the facility.

BACKGROUND

A six story chemical processing plant was originally designed and built in the late 1950’s. As with most buildings constructed during this era, the structural design was based on gravity and wind loading only. Over the building’s life span there have been numerous additions and modifications to the building layout. In the 1990’s the building was also seismically strengthened to meet the seismic requirements of the 1990 BOCA building code.

The 1990’s seismic upgrade was based on an earthquake with a 10% probability of exceedance in 50 years. However, new seismological information provides estimates of the earthquake that are larger than the 1990 BOCA code. The building was reviewed in 2012 as part of a post-Fukushima seismic walk down and the building was taken out of service for additional seismic upgrades.

In the intervening years, the design practice for commercial structures in the US shifted from a design basis earthquake with a 10% probability of exceedance in 50 years to an earthquake with magnitude of 2/3 times an earthquake with a 2% probability of exceedance in 50 years. Using the 2008 USGS hazard maps, this change alone resulted in an increase in seismic loads by a factor of 2.

Following the Fukushima event, there was considerable concern on how the building would respond to a beyond design basis event. The seismic margins assessment described in this paper was performed to address this issue.
The seismic margins assessment quantifies the seismic capacity by performing a median based pushover analysis with median member capacities and median deformation limits. Inelastic energy dissipation and the reduction in seismic input due to softening is accounted for by including the inelastic force reduction factor, $F_{\mu}$. The seismic capacity, expressed as a ground motion parameter, is convolved with the seismic hazard to develop the annual probability of failure, $P_F$, of the building.

**BUILDING DESCRIPTION**

The Chemical Processing Plant is a six story, steel-framed building with a base plan of 72’ in the North-South direction and 168’ in the East-West direction. The height of the building is 94’ and it has a basement below grade. Figure 1 shows an isometric view of the computer 3D model of the building. The original building was constructed in the late 1950, with multiple additions over the building’s life and a seismic upgrade in the 1990’s.

![Isometric View of the Finite Element Model Looking North-West](image)

Vertically, the floors and roof framing consist of pin-ended, simple span, beams connected primarily with web framing angles. The columns are typically three piece with splices above the third and fifth floors. On the east end of the building there is a two story truck bay and on the west end of the building there are bays with vertically offset floor framing and discontinuous floor diaphragms.

Laterally, loads acting on the floors are transferred by floor diaphragms consisting of checkered floor plates welded to the steel beams. The diaphragms transfer lateral loads to vertical braced frames which transfer the lateral load to the foundation. Due to the chemical process layout, the vertical bracing is limited to certain exterior bays and bays adjacent to the end bays, leaving the center portion of the building brace free.

The original construction utilized single angle cross bracing in the upper portion of the building and wide flange cross or single diagonal bracing in the lower portions of the building. The single diagonal braces were not installed in tension-compression pairs; thus, the building has unequal ultimate strengths depending on the direction of loading. Additionally, the centroid of the brace member typically intersects the column either above or below the floor girder resulting in eccentric column loads.

During the 1990’s seismic retrofit, diagonal tubular brace elements were added with welded gussets. As part of the 2013 seismic upgrade, additional diagonal brace elements were added. Some existing brace elements, numerous bracing connections, portions of the floor diaphragm, several columns and several of the column anchorages were also strengthened.
The steel braced system has an irregular bracing layout, horizontal eccentricities between mass and stiffness, vertical discontinuities in floor elevation and discontinuous floor diaphragms. Additionally, the use of single diagonal brace members causes the bracing to have different strengths in different directions. Given the complexities of this building’s load path, it is not possible to, a priori, determine the load direction which corresponds to the lowest building capacity. This complicated load path greatly increases the analytical task to quantify the seismic margins.

**EVALUATION BASIS EARTHQUAKE**

A seismic event with a 10% probability of exceedance in 50 years is selected as the evaluation basis earthquake (EBE). The seismic input used in this analysis is defined by the 2008 USGS Earthquake Hazards Program, which represented the best available data when the analysis was performed. The horizontal response spectrum for the EBE, shown in Figure 2, is extracted form the USGS hazard data for the building site. The spectrum corresponds to the local soil conditions, Site Class D.

Note that 2008 USGS does not define a vertical motion, and the vertical motion in this assessment is conservatively assumed to be equal to the lateral motion.

![Figure 2: Evaluation Basis Earthquake](image)

**ACCEPTANCE CRITERIA**

To prevent leakage of chemical components, the building must not collapse during or after a seismic event. Since aftershocks may occur after a seismic event, the building must also be capable of resisting aftershocks. The magnitude of the aftershocks is assumed to be equal to the magnitude of the primary seismic event.

To preclude collapse from both the primary seismic event and aftershocks, the lateral drift is limited to the story drift that results in the onset of significant strength degradation due to cyclic loading. This deformation limit corresponds to a loss of strength no greater than 10% of the strength used to define the lateral load capacity of the structure.

The lateral drift limit allows:

- The strength of a brittle component may be included in the development of the backbone curve capacity provided the lateral displacement is limited to account for the brittle component; or
• The strength of a brittle component may be neglected in the development of the backbone curve capacity. In this case the drift limit is based on a 10% loss in strength of the remaining credited members, not on the drift limit of the brittle component which was excluded from the capacity.

The first case may yield a higher static capacity with a lower drift limit, while the second case may yield a lower static capacity with a higher drift limit.

Deformation acceptance criteria consist of both story drift and individual brace element deformation limits. These deformation limits are used to define the median displacement capacity.

**Story Drift**

ASCE 43 establishes a permissible deformation limit of 0.02 times the story height for concentric steel braced frames at Limit State A, where Limit State A corresponds to “large permanent distortion short of collapse”. Limit State A is equivalent to this building’s no-collapse performance goal.

The Limit State A story drift in ASCE 43 is generally exceeded by 84% of the data. Assuming a coefficient of variation of 0.25 for the deformation data, the median inter-story drift ratio corresponding to collapse for a concentric braced frame is 0.0267. The pushover analyses are terminated at story drifts reaching 0.0267.

**Component Deformation Limits**

The bracing element tensile yield and compression buckling capacities have deformation limits and residual strengths based on the element backbone curves defined in ASCE 41. Typical permissible inelastic deformations in compression are 0.5 to 1.0 times the buckling deformation, depending on the brace slenderness ratio. In tension, the permissible inelastic deformations is 11 times the yield deformation. In typical bays with both tension and compression bracing, the compression bracing limits the allowable drift.

**SEISMIC MARGINS ASSESSMENT METHODOLOGY**

The seismic margins assessment quantifies the seismic capacity by performing a median based pseudo static pushover analysis with median member capacities and median deformation limits. The reduction in force due to inelastic energy dissipation and the reduction in natural frequency is included by considering an inelastic force reduction factor, $F_\mu$, on the median capacity. The seismic capacity, expressed as a ground motion parameter, is convolved with the seismic hazard to develop the annual probability of failure, $P_F$. The seismic margins assessment consists of the following steps:

First, a linear elastic analysis is performed to determine the inertial nodal forces due to the EBE. These inertial forces are incrementally increased and applied to a nonlinear pushover analysis model and the building’s lateral drifts for each increment of the loading are determined. This process is continued until the best estimate story drift corresponding to unacceptable performance is reached at any location in the structure. The resulting scale factor on the EBE is defined as the best estimate or median capacity factor, $F_C$.

Next, a best estimate inelastic force reduction factor, $F_\mu$, is determined as described in the following sections. This factor defines the reduction in seismic demand that results from a reduction of the effective natural frequency of the structure and increase in energy dissipation due to nonlinear cycles of drift.

The best estimate seismic margin scale factor, $F_M$, is defined by the product of $F_C$ and $F_\mu$, given by Equation 1 as:

$$F_M = F_C \times F_\mu$$  \hspace{1cm} (1)

The pushover analysis assumes that the nonlinear behavior is limited to brace elements. The remaining items in the seismic load path are evaluated using the demands from the pushover analysis to ensure that these components have sufficient strength to develop $F_M$. 
Elastic Analysis

An elastic analysis was performed to characterize the building’s dynamic response and to develop a seismic load vector. The seismic mass is consistent with ASCE 43. The elastic response spectrum analysis is performed using the modal combination rules in NRC Regulatory Guide 1.92. The finite element model shown in Figure 1 is used to perform both the elastic and pushover analyses.

The building’s natural frequencies are 1.4 Hz in the North-South direction and 1.6 Hz in the East-West direction. The building’s mode shapes indicated that some of the lateral modes are coupled, which is expected for this structure with irregular mass and lateral stiffness. Vertically, the framing consists of simple spans and coupling between the vertical and lateral modes was not observed. Examining the mode shapes helped to identify components with problematic load paths, which were subsequently strengthened.

The seismic load vector is the product of the absolute acceleration times the mass at each node in the building. Inertial nodal forces are determined by assuming that all of the peak accelerations are in phase and occur at the same time. Separate load vectors are developed in each of the three orthogonal directions.

Member Capacities

Median member capacities of structural steel members and connections are determined following the AISC 360-05 specifications with median material properties and $\phi=1.0$. In a couple of members where AISC capacities have additional conservatism, the additional conservatism is removed by comparing the predicted capacity using the code equations to median test data.

Median steel material properties, yield strength and ultimate tensile strength, are based on ASCE 41. The median concrete compressive strength includes the ratio of median to design strength (1.2) and the effects of concrete aging (1.2) for a median compressive strength of 1.44 times the minimum specified design strength.

Comparing brittle and ductile failure modes with similar median capacities, the brittle failure mode is assumed to dominate unless the brittle failure mode has a capacity greater than or equal to 125% of the ductile failure mode capacity. When bracing connections are strengthened, the capacity of brittle failure modes, such as bolt shear or weld fracture, are increased until the connection capacity is 125% of the member yield capacity.

Pushover Analysis

In the 3D building model, selected brace elements are represented by nonlinear spring elements, where the typical nonlinear load-deformation curve is shown in Figure 3. In the figure, $P_C$ is the median buckling capacity, which is based on AISC specifications, $\Delta_C$ is the buckling deformation, $T_Y$ is the yield capacity, $\Delta_Y$ is the yield deformation, and $\Delta_{Plastic}$ is the permissible inelastic deformation.

The pushover analysis is preformed in two steps with the first step being the application of gravity loads and the second step being the pseudo-static incremental application of seismic loads. This analysis is performed using Abaqus with the Riks solver and considers both element (nonlinear spring) and geometric (P-$\Delta$) nonlinearities.

Modern finite element codes such as Abaqus have the ability to model tensile yielding, compression buckling and the complexities of post buckling hinge formation and the resulting compressive strength reduction. However, simple nonlinear springs are used in this analysis to represent the complex nonlinear member behavior shown in Figure 3. A key advantage of nonlinear springs is that the capacity and deformation limits can be defined using AISC 360 and ASCE 41 and the analysis can focus on building behavior. A limitation of the nonlinear springs are that they load and unload on the same curve and are only appropriate for the first quarter cycle of loading, which defines a pushover analysis.
Spatially, seismic loads are combined using the ASCE 4 100-40-40 rule. Rigorously applying the 100-40-40 rule yields 24 seismic load combinations. Given the existing engineering studies which evaluate lateral and vertical demand to capacity ratios, it is judged that lateral seismic loads will limit the seismic margins of this building. Additionally, vertical seismic loads acting on beams will not control the seismic margins. However, an upwards vertical load acting on a column anchorage is judged to be limiting compared to the downward vertical load because column anchorage typically has a yield load that is smaller than the column compression capacity. Note that, the upwards vertical force acting on the column is moderate compared to the uplift force from the bracing overturning. Thus, the seismic load combinations are reduced to the eight load combinations in Equation 2.

\[
E_X = \pm 1.0E_{EW} \pm 0.4E_{NS} - 0.4E_V \\
E_Y = \pm 0.4E_{EW} \pm 1.0E_{NS} - 0.4E_V
\] (2)

The vertical seismic load is selected to generate realistic column reactions instead of the peak vertical load acting simultaneously on each beam. This is done by considering a portion of the vertical mass acting at the beam support, the remaining mass acting at the beams center and beams on different floors vibrating out of phase because they have different natural frequencies.

Recall that this building has mass and stiffness eccentricities, coupled lateral response and different lateral strengths in different directions due to the use of single diagonal braces. Thus, eight separate pushover analyses are performed, one for each of the eight load combinations in Equation 2. The results of the pushover analyses with the lowest capacity are used to represent the building’s seismic capacity.

**INELASTIC FORCE REDUCTION FACTOR**

The system ductility factor, \( \mu \), is determined following the approach presented in EPRI NP-6041-SL. The nonlinear deformed shape of the structure corresponding to the maximum permissible distortion being reached in the critical location of the structure is defined by the nonlinear pushover analysis. The system ductility, \( \mu \), is estimated from Equation 3.

\[
\mu = \frac{\sum W_i \delta T_i}{\sum W_i \delta e_i}
\] (3)

where \( W_i \) are the inertial weights applied at each story of the structure and \( \delta T_i \) is the total displacement of each story corresponding to the permissible total story distortion occurring in the critical story. The dis-
The system ductility factor in Equation 3 is generalized for nodal loads and displacements by substituting nodal values for each i, and summing over the total number of nodes.

Given \( \mu \), the inelastic factor, \( F_\mu \), is determined using the effective frequency, effective damping (EFED) approaches presented in NUREG/CR-3805. Of the three EFED models that are presented in NUREG/CR-3805, the Iwan Model and the Sozen Model are applicable for both Bilinear and Takeda model hysteresis behavior which have either no pinching or only small pinching of the hysteresis loops during nonlinear cyclic behavior. These two models are applicable for steel and properly reinforced concrete moment frame behavior.

A third model in NUREG/CR-3805 was specifically developed to be applicable for severely pinched hysteresis behavior associated with nonlinear cyclic behavior of shear walls. Based on time-history analyses performed using a braced frame hysteresis model, a recommendation is made to correct the shear wall model for braced frame hysteresis behavior.

For this building, \( F_\mu \) estimates are made using each of the following; NUREG/CR-3805 Model for each of 4 strong motion duration ranges; NUREG/CR-3805 Model, with the recommended modification for braced frames; the Iwan Model, which is potentially unconservative for braced frames and additionally the Newmark and Hall elasto-plastic response model, which over-estimates the energy dissipation compared to a pinched response model, is used as a check on the inelastic factor.

Comparison of these results for the moderate system ductilities in this analysis, shows that the computed \( F_\mu \) is only moderately different for each of these four EFED Models. Of the four models, the NUREG/CR-3805 recommendation for brace frames is the most appropriate and this model is used to compute the building’s inelastic factor.

**PROBABILITY OF FAILURE**

The mean seismic risk \( P_F \) (Annual Probability of Failure) can be obtained by numerical convolution of the mean seismic hazard curve and mean fragility curve by:

\[
P_F = \int_0^{+\infty} \left( \frac{dH(a)}{da} \right) P_{F/a} \, da
\]

where \( P_{F/a} \) is the conditional probability of failure given the ground motion level, \( a \), which is defined by the mean fragility curve, and \( H(a) \) is the mean hazard exceedance frequency corresponding to ground motion level, \( a \). The mean fragility curve is typically defined by a lognormal distribution with a median capacity \( C_M \) and a natural logarithmic standard deviation \( \beta \). For a wide variety of structures, \( \beta \) typically ranges from 0.3 to 0.5 with an average value of 0.4.

For this building, the annual probability of failure is best defined by convolving the 1 Hz Spectral Acceleration fragility estimate with the 1 Hz seismic hazard because as the building goes inelastic its effective frequency drops to about 1 Hz.

**ANALYSIS RESULTS**

A typical load-deformation or backbone curve is shown in Figure 4 which corresponds to the 100% South + 40% East loading. The horizontal axis corresponds to southern deformation of the sixth floor and the vertical axis corresponds to a scale factor, \( F_C \), on the EBE loading. Each horizontal line on the figure corresponds to a change in state of one of the brace elements, where state changes correspond to either yielding in tension,
Table 1: Pushover Results

<table>
<thead>
<tr>
<th>Load Case</th>
<th>$F_C$</th>
<th>$\Delta C$</th>
<th>$\mu$</th>
<th>$F_{\mu}$</th>
<th>Median Capacity, $F_M$</th>
</tr>
</thead>
<tbody>
<tr>
<td>100% East + 40% North</td>
<td>1.76</td>
<td>6.22</td>
<td>2.60</td>
<td>1.96</td>
<td>3.45</td>
</tr>
<tr>
<td>100% East + 40% South</td>
<td>1.65</td>
<td>4.83</td>
<td>2.08</td>
<td>1.75</td>
<td>2.90</td>
</tr>
<tr>
<td>100% West + 40% North</td>
<td>1.35</td>
<td>-4.49</td>
<td>2.58</td>
<td>1.95</td>
<td>2.63</td>
</tr>
<tr>
<td>100% West + 40% South</td>
<td>1.34</td>
<td>-4.51</td>
<td>2.58</td>
<td>1.95</td>
<td>2.62</td>
</tr>
<tr>
<td>100% North + 40% East</td>
<td>1.72</td>
<td>9.66</td>
<td>2.32</td>
<td>1.82</td>
<td>3.13</td>
</tr>
<tr>
<td>100% North + 40% West</td>
<td>1.68</td>
<td>9.17</td>
<td>2.29</td>
<td>1.81</td>
<td>3.05</td>
</tr>
<tr>
<td>100% South + 40% East</td>
<td>1.50</td>
<td>-5.74</td>
<td>1.92</td>
<td>1.67</td>
<td>2.51</td>
</tr>
<tr>
<td>100% South + 40% West</td>
<td>1.52</td>
<td>-6.33</td>
<td>2.03</td>
<td>1.72</td>
<td>2.60</td>
</tr>
</tbody>
</table>

buckling in compression or unloading to post-buckled residual capacity. As seen in the figure, the first state change occurs at $1.01xEBE$ and is due to buckling of a brace member. The median capacity for this loading is $F_C=1.50$ times the EBE.

For compression members, the median capacity for buckling is roughly 1.3 times the code capacity. Thus, the median building capacity, $F_C$, is roughly $1.5 \times 1.3 \approx 2$ times the capacity at the first code exceedance. This is also known as the over-strength factor.

The lateral drift from four different locations is shown in Figure 4. The difference in deformation between the two ends of the building, represented by Gridlines B02 and A6, indicate that the building is twisting as it deforms laterally. At a lateral deformation of 4.3 inches, a slight ($\approx 3\%$) reduction in strength is observed in the Gridline A6 deformation, which is caused by a buckled brace unloading. This reduction in capacity is recovered as the load is further redistributed to other members.

Analysis results for the each of eight different loading cases are summarized in Table 1. As discussed above, $F_C$, is the median capacity from the pushover analysis which ranges from 1.34 to 1.76. The median seismic capacities range from 2.51 to 3.45 times the EBE.

The lowest median seismic capacity, $F_M=2.51$, is due to the 100% South + 40% East loading, which has a capacity factor, $F_C=1.50$ and a system ductility 1.92. The 100% West + 40% South has a lower capacity...
factor, $F_C=1.34$ and a higher system ductility of 2.58 which yields a larger median seismic capacity of $F_M=2.62$. From a practical viewpoint, seismic capacity factors of 2.51 and 2.62 are essentially equal.

The building fragility curve has a median capacity, $C_M=Sa(1\text{Hz})\times F_M$ and a lognormal standard deviation, $\beta$, of 0.3, 0.4 or 0.5. The fragility is convolved with the seismic hazard using Equation 4 to yield the annual probability of failure.

As discussed above, the first state change, member buckling, occurs at $F_C=1.01$. Up to this point the response is linear, $F_M=1.0$ and the corresponding seismic capacity is $F_M=1.01$. Similarly, the first member yielding occurs at $F_C=1.12$, again with nearly linear response, $F_M\approx1.0$, and $F_M=1.12$.

If the structure lacks ductile detailing, the connections are not able to develop the full members capacity and the seismic capacity of the building would be, at most, $F_M=1.01$ to 1.12. Comparing the probability of failure with $F_M=1.01$ and to the probability of failure with ductile behavior, $F_M=2.51$, indicates that ensuring a moderate level of ductile response reduces the probability of failure by a factor of roughly 3.

Thus, strengthening the connections to obtain a moderate level of ductile behavior, with a system ductility of 1.9, results in a significant reduction in risk.

Note that structural steel design in the nuclear industry, using ANSI/AISC N690, is often based on elastic analysis concepts and may neglect inelastic response considerations. These structures could also have a similar risk reduction if the structures are capable of moderate amounts of ductility. In contrast, ASCE 43 focuses on meeting probabilistic performance goals, which includes beyond design basis earthquakes and requires ductile detailing for both steel and concrete structures.

**CONNECTION EVALUATIONS AND MODIFICATIONS**

The median member capacities used in the pushover analysis are the brace members tension yield and compression buckling capacities. These capacities are based on the assumption that the connections are capable of developing these capacities. Extensive work was required to evaluate each connection in the seismic load path.

The one advantage of performing pushover analyses for each of the eight loading permutations in Equation 2 is that the full range of diagonal brace forces is known for each of the brace members. This provides a technical basis to not upgrade connections of braces that did not have large demands. An alternative approach would have been to impose the ductile detailing requirements of AISC 341 on all the bracing connections. This costly alternate approach would have resulted in significant additional facility modifications without a corresponding reduction in seismic risk.

The connection capacities are compared to 125% of the enveloping demands from the eight pushover analyses to ensure that the connection is stronger than the member. The connections with a capacity less than 125% of the demand were strengthened. A total of 53 brace members required connection strength upgrades.

Additionally, portions of the floor diaphragm and several collector beams required strengthening. Several columns required strengthening to resist eccentric bracing loads and bending caused by vertical offsets in the load path. The anchorage capacity of several columns also had to be upgraded.

**CONCLUSION**

A 1950’s vintage chemical process structure was reviewed in 2012 as part of the post-Fukushima seismic walkdowns and subsequently taken out of service for seismic upgrades. The upgrades included strengthening the building to meet licensing requirements supplemented by current seismic hazard data and a seismic margins assessment to quantify the facility’s median seismic capacity.
The seismic margins assessment included a nonlinear pushover analysis of the facility which determined a seismic margin scale factor based on the lateral drifts of the facility up to when unacceptable performance is reached at any location in the structure. Based on the nonlinear pushover analyses, the weak links of the structure in terms of connections, members, collectors, etc. were identified.

Extensive connection upgrades were required to develop member capacities. The connection upgrades allowed a system ductility, $\mu=1.9$. Providing the connection upgrades and ductility resulted in a factor of 3 reduction in risk.

Current building codes contain seismic detailing requirements which are intended to develop the full expected member capacity and provide ductility. Applying these requirements to an existing building would have resulted in a significant increase in modification costs. Using the pushover analyses to identify the buildings weak links provided a technical basis for strengthening critical members alone, and indicated that strengthening additional members and connections would not result in an additional risk reduction.

Several lessons were learned during this seismic margins assessment:

- Significant reductions in seismic risk are possible even in an existing 50 year old building with mass and stiffness irregularities.
- Adequate connection strength is essential to allowing the redistribution of forces and to maintain capacity while the structure deforms nonlinearly.
- Ductile detailing rules ensure that the connection strength is adequate to develop the members capacity. Including ductile detailing in new structures ensures that they have sufficient strength to deform inelastically.

The seismic margins assessment results were reported to the regulatory agency and were a positive factor in the decision to resume operations at the facility.

REFERENCES


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ASCE 41-06, Seismic Rehabilitation of Existing Buildings, American Society of Civil Engineers, 2006.

